REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAM-COLUMN JOINTS

HONORS THESIS

Presented in Partial Fulfillment of the Requirements to Graduate with Distinction from the Civil and Environmental Engineering and Geodetic Science Department and with Honors from the College of Engineering at The Ohio State University

By

Seth T. Salisbury

○○○○○○

The Ohio State University

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Undergraduate Honors Examination Committee: Approved by:

Dr. Halil Sezen, Advisor

Dr. Shive K. Chaturvedi

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Advisor

Civil Engineering Honors Undergraduate Program

ABSTRACT

This research study investigates a method for strengthening broken or cracked reinforced concrete beam-column joints. These specimens were previously tested by Fisher (2009). The restoration method used for this study involved repair of cracks using non-shrink grout and application of fiber reinforced polymer (FRP) over the surface of three specimens. First, non-shrink grout, a high compressive strength concrete mix, is used to fill in the cracks of the beam-column joint specimens. Then, the FRP is glued to the surface of the joints using a high strength epoxy material. Specimens are tested under cyclic loading until failure. This restoration method was effective in increasing the capacity of the damaged specimens.

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VITA

September 8, 1987……………………………Born-Portsmouth, Ohio

June 13, 2010…………………………………B.S. Civil Engineering

FIELDS OF STUDY

Major Field: Civil Engineering

Specialization: Structural Engineering

Minor Field: Surveying & Mapping

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CHAPTER 1: INTRODUCTION

1.1 Overview

Concrete is a composite material made of cement, water, sand, and course aggregates. Cement is a fine powder that begins to harden by chemical reactions after water is added to it and binds the sand and aggregates together as it hardens. Usually, after 28 days from the time the cement and aggregates have been exposed to water or hydration, the concrete develops its approximate full strength. Additional strength will continue to develop over time; however, this additional strength can usually be ignored for calculations since it is relatively small as compared to the 28-day strength.

Due to its physical properties, concrete is much stronger in compression (pushing) than in tension (pulling). Because of this phenomenon, pure concrete structural members cannot be subjected to large tensile stresses and need another material to carry tensile loads. Usually, this material is deformed steel rebar and is encapsulated by the concrete. Rebar is made with different size diameters, and usually, a number is used to correspond to the number of eighths of an inch diameter size.

A beam is a member that carries mainly forces perpendicular to its longitudinal axis and very small or no axial load. A column is a member that carries significant large axial loads, at least 10% or more of its axial capacity. When a beam and column are interconnected and poured simultaneously, the location of the intersection of the beam and column is a beam-column joint. The response of this joint is extremely important because there are several different types of loads resisted at this location. Usually, the critical types of loads are shear forces and bending moments. Shear forces are internal forces perpendicular to the longitudinal axis of the column created by external loadings. Bending moments are internal forces acting in the column and beam ends caused by external loadings. Therefore, because of these different types of loads and their relatively large values and interaction, this region of the concrete beam-column may crack or fail first if the beam is designed to be stronger than the beam-column joint region.

This research project focuses on restoring the strength and deformation capacity of previously damaged beam-column specimens. This report begins by explaining the scope and objectives of the project, continues with literature review, then discusses the actual experimental design, and ends with research results and an interpretation of those results.

1.2 Scope and Objectives

This project involves retrofitting three reinforced concrete beam-column joints that Fisher (2009) had built and tested to failure. Repair and retrofitting concrete beam-column joints involves removing any loose or weak concrete near the joint, building forms so new concrete or grout can be poured in clean holes and gaps, placing epoxy and fiber reinforced polymers in areas on the beams and columns that is most critical for load resistance, testing the retrofitted concrete beam-column joints, and comparing original and retrofitted strengths to determine if the beam-column joints gained additional strength through retrofitting. The main objective is to determine if the proposed retrofitting method provides additional strength as compared to the strength of the original specimens. An example of FRP being used is shown in Figure 1.1.

Figure 1.1-Example of FRP used in Tuttle Parking Garage on the campus of The Ohio State University, Columbus, Ohio

1.3 Literature Review

Engindeniz et al. (2005) completed a study of different types of retrofitting, one of which was FRP (Figures 1.2 and 1.3). However, the FRP used in their study was glass fiber reinforced polymer (GFRP) as well as carbon reinforced polymer (CFRP). CFRP was used in this research project, and this project used different wrapping techniques.

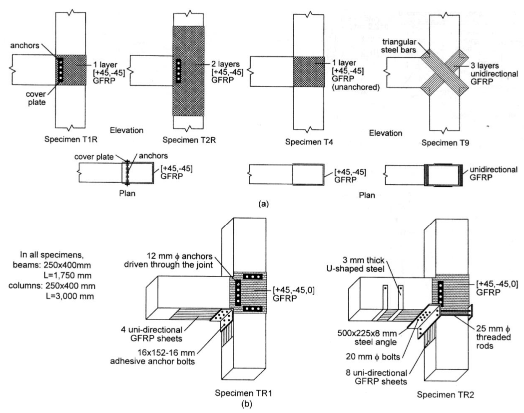

Fig. 9—Glass fiber-reinforced polymer-strengthened specimens tested by: (a) Ghobarah and Said,[30] and (b) El-Amoury and Ghobarah.[37]

Figure 1.2-Possible wrapping techniques presented in Engindeniz et. al (2005)

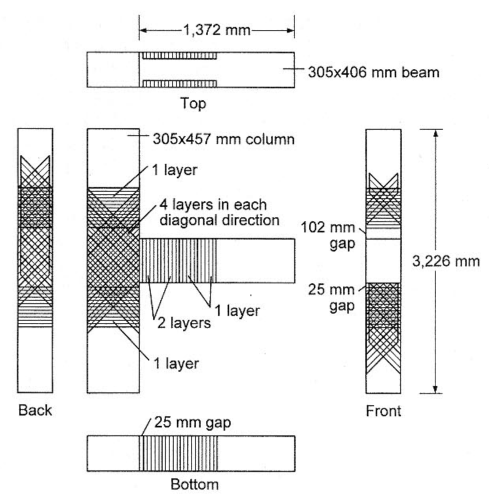

Fig. 10—Carbon fiber-reinforced polymer-strengthened specimen tested by Clyde and Pantelides.[38]

Figure 1.3-More wrapping techniques presented in Engindeniz et al. (2005)

Ghobarah and Said (2001) completed a research project using GFRP to re-strengthen damaged beam-column joints (Figure 1.4). This project used previously proposed and new ideas for retrofitting the beam-column joint. Ghobarah and Said (2001) found that GFRP did add strength back to beam-column joints, and the specimens failed in flexure.

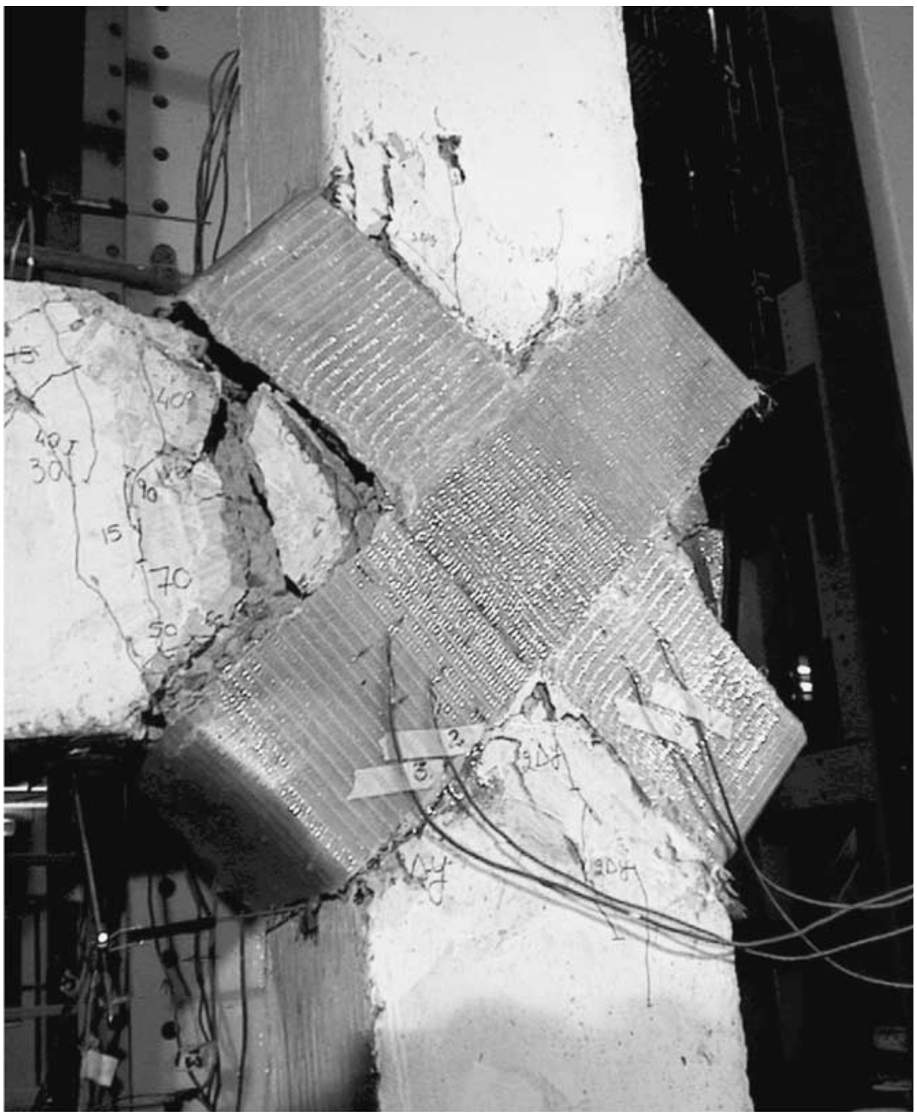

Figure 1.4-Beam-column joint wrapping techniques (Ghobarah and Said 2001)

Mahini and Ronagh (2009) completed a research project involving using CFRP on beam-column joints to retrofit the joints (Figure 1.5). They found that FRP did restore some strength back to broken beam-colums.

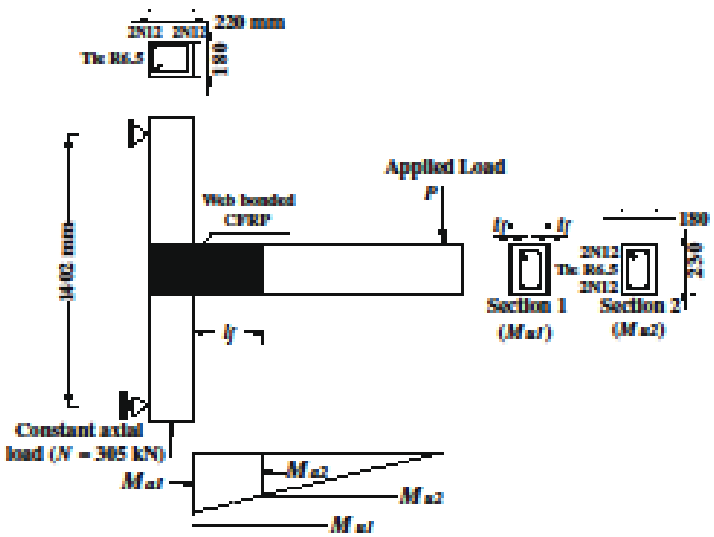

Fig. 1. Retrofitted specimens reported by Mahini [10].

Figure 1.5-More beam-column joint wrapping techniques (Mahini and Ronagh 2009)

Orton et al. (2008) studied the behavior of CFRP attached to the beam surface, and how it de-bonds from the concrete surface. They came up with an anchoring method to keep the CFRP attached to beams while loading. They tested 40 specimens and found that using several small anchors was the best method of keeping the CFRP attached to the beams.

Adin et al. (1993) completed a research project retrofitting specimens using epoxy injection on four damaged specimens. The epoxy was pressurized and was pumped into small cracks within the specimens causing the cracks to be sealed. This is another area of research that could be studied.

CHAPTER 2: RETROFIT DESIGN AND APPLICATION

2.1 Introduction

This chapter provides an overview of original design of the concrete beam-column joints (Fisher 2009), the retrofitting method used to repair and restore strength to Fisher's broken concrete beam-column joints, and the testing apparatus and procedure used to test the retrofitted beam-column joints.

2.2 Concrete Beam-Column Design

This section discusses the design details of the beam-column joint specimens, rebar reinforcement used, approximate 28-day concrete compressive strength, and other experiment parameters determined by Fisher (2009). For more details about the design of the concrete beam-columns, please see Fisher (2009).

2.2.1 Steel Reinforcement Specifications

The Table 2.1 contains the steel reinforcement details used in the beam-column joint specimens tested by Fisher (2009). Fisher tested six joint specimens. In this study, only three specimens were used since their columns were reinforced with standard longitudinal and traverse steel reinforcement. All steel rebar consisted of standard ASTM A615 Grade 60 deformed bars. All beams' rebar were tied with 7 in. by 5 in.

rectangular closed stirrups with 135 degree end hooks, using ¼ in. diameter smooth bar. Spacing of column ties and beam transverse reinforcement varied (Table 2.1). Columns were tied using 5 in. by 5 in. closed traverse reinforcing hoops (Fisher 2009).

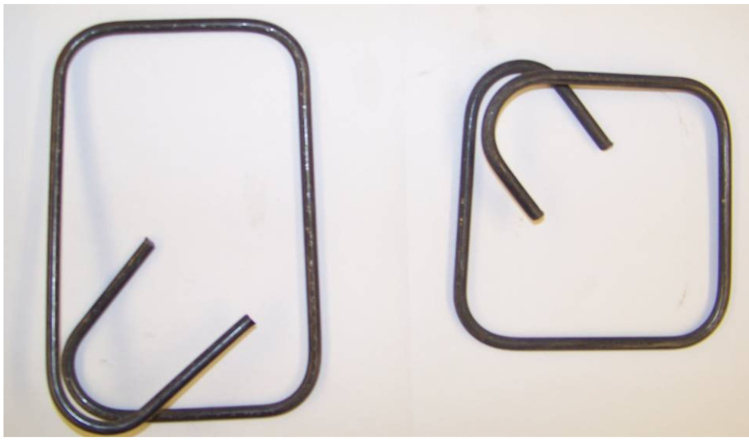

Figure 2.1-Tranverse reinforcement used to tie beam and column, respectively (Fisher 2009)

Table 2.1-Test Specimen Specifications (Fisher 2009)

Specimen Number	Specimen Name	Column Reinforcement		Beam Reinforcement	
		Method	Tie Spacing (in.)	Longitudinal (top & bottom)	Tie Spacing (in.)
1	C-2-RC	Rebar	1.5	3 - #3	1.5
3	E-1-RC	Rebar	2.5	2 - #3 & 1 - #4	3.5
5	B-1-R	Rebar	2.5	3 - #4	1.75

2.2.2 Specimen Concrete Strength

Table 2.2 and Figure 2.2 show the approximate strength of the concrete used for the original specimens.

Table 2.2-Concrete Cylinder Test Data (Fisher 2009)

Test Date	13-Sep	20-Sep	27-Sep	4-Oct	7-Nov	21-Nov
Time (days)	7	14	21	28	62	76
Sample #1 (kips)	24.9	34.1	37.3	43.1	52.9	*Projected
Sample #2 (kips)	25.3	34.6	38.9	42.9	49.5	
Sample #3 (kips)	27.3	28.0	38.3	44.2	52.9	
Average (kips)	25.80	32.22	38.17	43.40	51.77	
Average (psi)	2053	2564	3037	3453	4119	4330

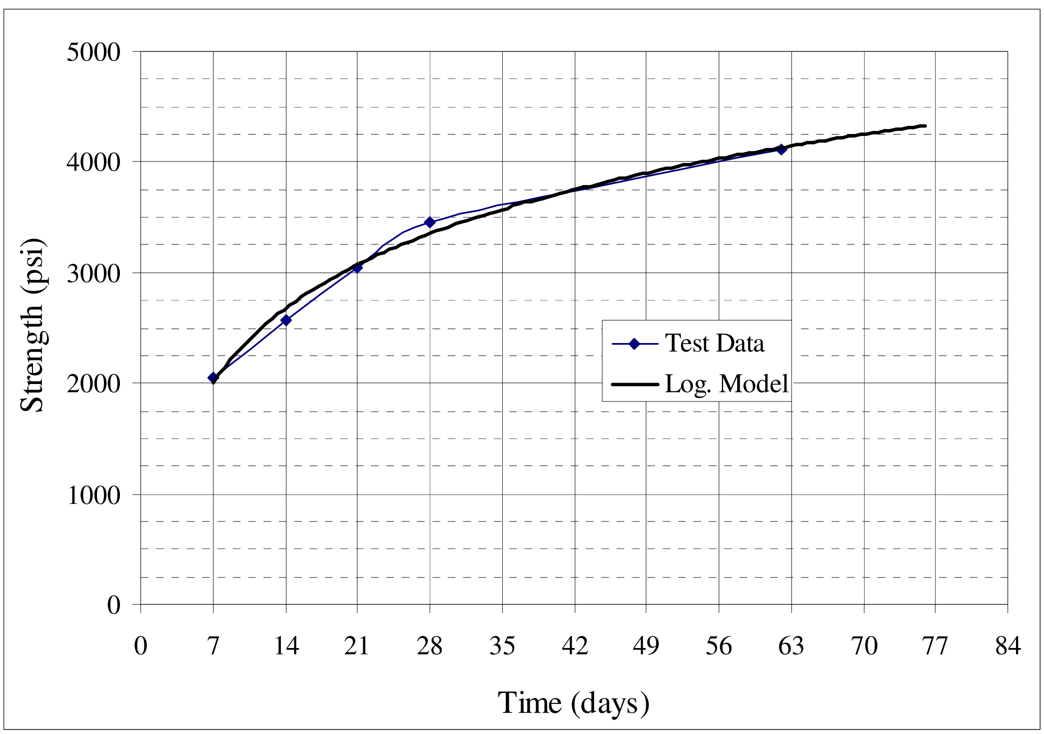

Figure 2.2 – Graph of Concrete Test Cylinder Strength Over Time (Fisher 2009)

2.2.3 Beam-Column Specimen Dimensions

The beams were 6 in. wide by 8 in. deep rectangular sections with a length of 20 in. from the face of the column. The columns were 6 in. square sections with a height of 30 in. Dimensions were selected in order to represent an approximately 1/3 scale of average sized beam-column joint and were limited by the dimensions, maximum load and deflection capacity of the existing testing frame (Fisher 2009).

2.3 Retrofitting Method

In this section, the methods used to retrofit Fisher's specimens are discussed. Material properties used to retrofit the beam-columns described. Figures are included to help describe the actual processes used to retrofit the specimens.

2.3.1 The Beginning Specimens

This project began with Fisher's finished project. Fisher (2009) tested six beam-column joint specimens to failure. Three of these specimens' (Specimens 1, 3, and 5) columns were reinforced with standard longitudinal and traverse steel. The remainder specimens' columns were reinforced with a prefabricated cage system (PCS), a special reinforcement recently developed at The Ohio State University (Figure 2.3). Due to the different type of reinforcement, only specimens 1, 3, and 5 were considered in this research project since this is the traditional reinforced concrete system currently used.

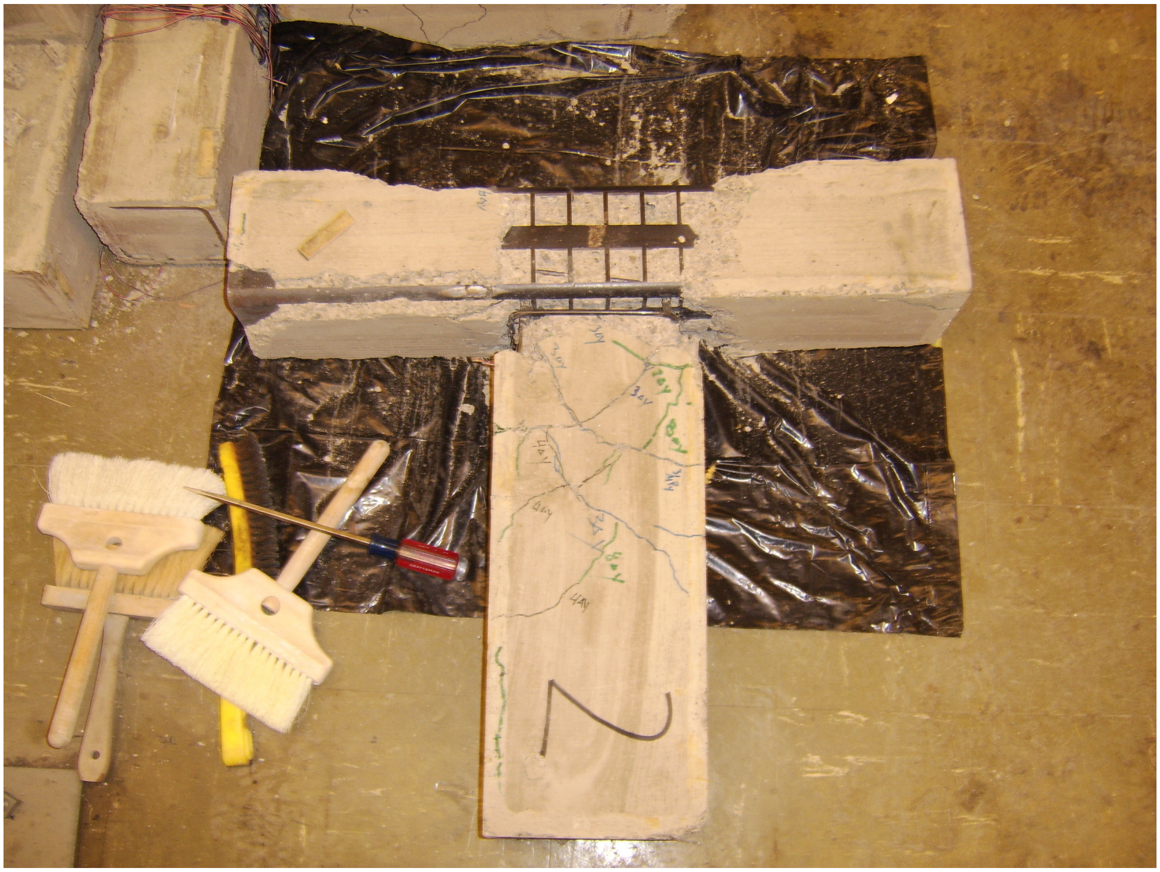

Figure 2.3-Fisher's Specimen 2 showing the typical damaged beam-column joint (Note: This specimen was not retrofitted; this is only to show a typical damaged specimen.)

2.3.2 Form Building and Grouting

The first step was to clean and prepare the three damaged specimens tested by Fisher (2009). Chisel and hammer were used to clean any loose pieces of concrete that may have been still attached to the specimens. Then, the next step was to take the old formwork used by Fisher to make new formwork around the highly damaged beam-column joints (Figures 2.4 and 2.5).

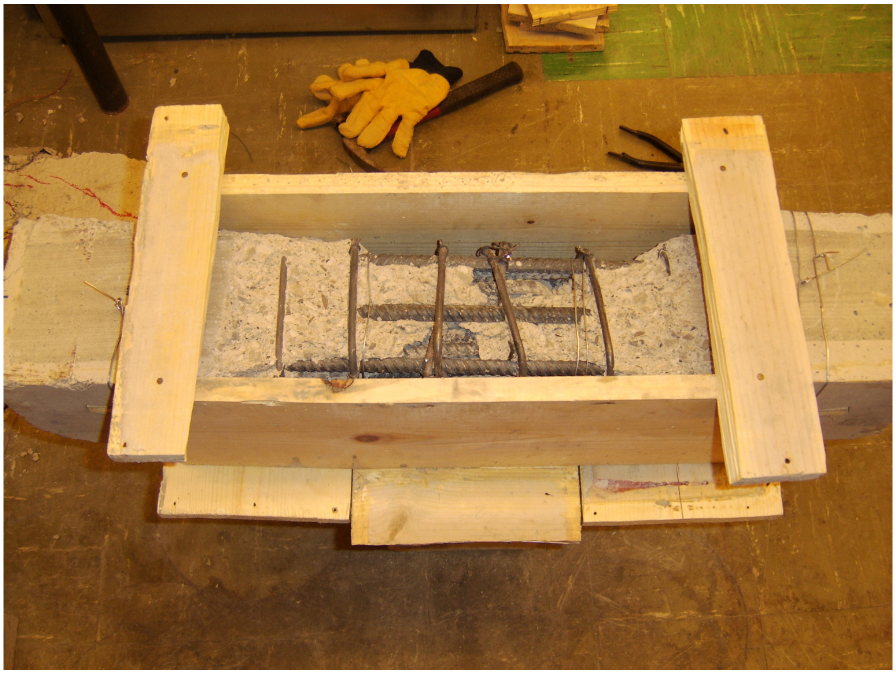

Figure 2.4-Specimen 5 forms from top. Back of column is shown.

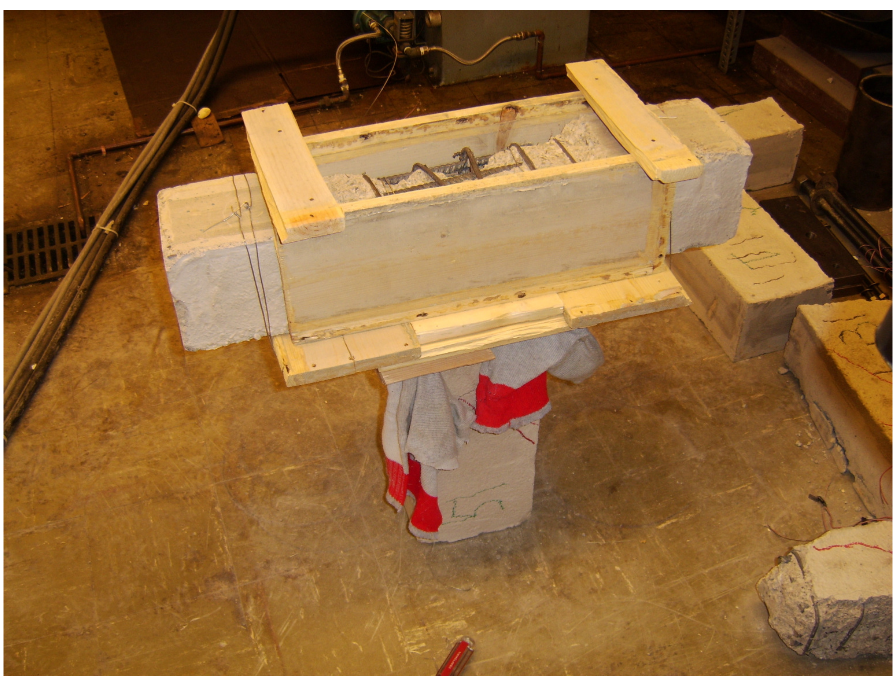

Figure 2.5-Specimen 5 forms from side

All specimens had a similar form-making process for their beam-column joints. However, Specimen 3 required additional formwork for one of its damaged column ends (Figure 2.6).

Figure 2.6-Specimen 3 extra formwork for damaged end (side view)

Figure 2.7-Specimen 3 damaged column end

Once the formwork was completed, a high-strength non-shrink grout was used to replace the missing concrete. This grout is made by QUIKRETE® (Figure 2.8). Attached in the Appendix is the specification data for this product.

Figure 2.8-Bag of QUIKRETE® grout

Approximately one gallon of water was added to a 50-pound of grout and mixed in a mixing box for five minutes before being added to the beam-column joints (Figures 2.9). After mixing the water and grout for five minutes, the beam-columns' areas that were to be grouted were coated with water to help the grout's chemical reaction and bonding to the original concrete. The damaged areas of beams and columns were sprayed with water prior to application of grout (Figure 2.10)

Figure 2.9-Grout mixed with water after 5 minutes

Figure 2.10-Water-coated original concrete to be grouted

After coating the original concrete with water, the mixed grout was placed on the areas to be repaired. The grout was rodded to ensure that the grout penetrated and covered all of the exposed original concrete. Then, the grout was smoothed to form a smooth cover (Figures 2.11 and 2.12).

Figure 2.11-Smoothing the grout

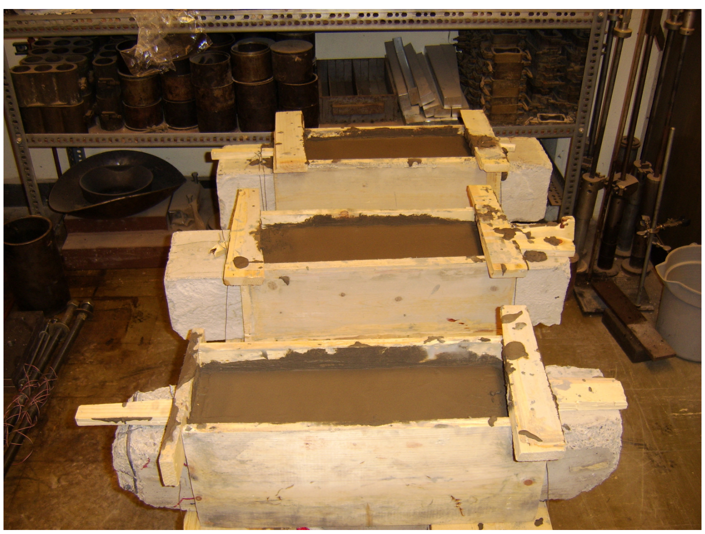

Figure 2.12-The finished grouted joints

Once the joints were grouted, the grout was covered with plastic bags to allow the grout to cure properly (Figure 2.13). Also, for three days after grouting was completed, water was sprinkled over the grout to allow proper curing. This occurred twice a day. The grout is projected to have a compressive strength of 14,000 psi after 28 days.

Figure 2.13-Curing procedure used

After the grout was allowed to cure for one week, the forms were removed. A rubbing stone was used to smooth the semi-cured grout to a level even with the existing concrete (Figure 2.14). Also, a hammer and chisel were used to break any loose grout that would not provide any structural strength.

Figure 2.14-Using the rubbing stone to smooth rough grout

Also, places that were still missing concrete and grout were puttied by hand using the non-shrink grout and without creating forms (Figures 2.15 and 2.16). Again, the specimens were covered with plastic to allow for curing. Also, they were watered again two times a day for three days.

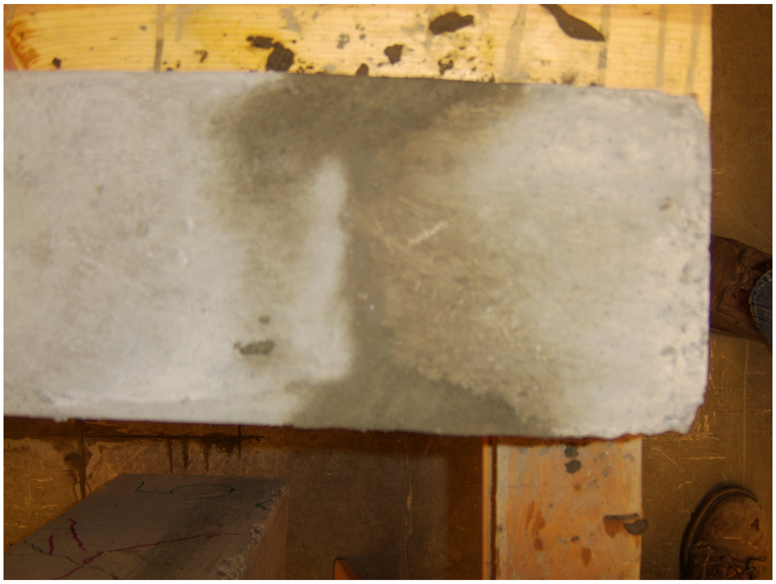

Figure 2.15-An example of a hand-grouted region

Figure 2.16-Another example of a hand-grouted region

2.3.3 Spackling

Two weeks after the last hand-grouted region was completed, a coat of spackling was applied to areas where the FRP was to be wrapped. The spackling was added to smooth the rough grouted edges. Thus, the forces within the FRP would be distributed evenly over more area instead of jagged edges (less area for the forces to act) creating stress concentration within the FRP wrap. Less stress will allow the FRP to withstand more force before breaking. Any spackling found at a local hardware store could be used (Figure 2.17).

Figure 2.17-Spackling used for smoothing rough edges

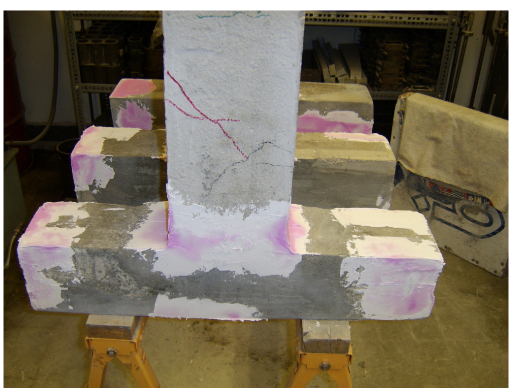

Figure 2.18-Specimens after spackling complete

2.3.4 FRP Wrap Application

The beam column joints were wrapped with the FRP in three different areas: longitudinally along the top and bottom of the beam's sides, around the four faces of the top and bottom of the column in the joint region, and around the beam-column joint in a "X" pattern (Figure 2.19). These places where the FRP was placed were designated as the most critical sections of the specimens based on experimental data reported by Fisher (2009). Thus, these places needed extra reinforcement to withstand more loading.

Two different types of carbon FRP were used due to material availability: SikaWrap® Hex 117C and SikaWrap® Hex 230C. The 117C Wrap was used longitudinally along the top and bottom of the beam's sides. This wrap was 3 in. wide by 0.02 in. thick and was wrapped 12 in. past the face of the column on each side of the beam as one layer. However, only 2 in. were considered when doing calculations due to the sides of the FRP fraying and not providing any structural strength. The 230C Wrap was used around the perimeter of the ends of each column two times. The wrap at the ends of the column was 3 in. wide (Figure 2.20). Also, the 230C Wrap was used to wrap the beam-column joint in an "X" shape one time. Again, the wrap was 3 in. wide. Attached in the Appendix is the product data sheet for each type of FRP wrap. Also, attached in the Appendix are sample calculations of determining the approximate strength of the FRP. These calculations involved finding the available tensile load in the FRP cross-section by taking the tensile strength of the FRP and multiplying that by the cross-section area. Then, the available moment was calculated by taking the available tensile load and multiplying by the distance from the edge of the beam to the centroid of the FRP wrap. Then, this available moment was divided by the distance from the face of the column to the load cell (approximately 18 in.).

$F_{FRP} = 2 * f_{FRP} * w_{FRP} * t_{FRP}$ Note : '2' comes from FRP being on each side of the beam.

$M_{FRP} = F_{FRP} * (d - 0.5 * a)$

$M_{Total} = M + M_{FRP}$

$P_{Load} = M_{Total} / (18 \text{ in.})$

where :

F_{FRP} = the available tensile load in the FRP

f_{FRP} = the available tensile strength of the FRP

w_{FRP} = the width of the FRP

t_{FRP} = the thickness of the FRP

M_{FRP} = the available moment from the FRP

d = the distance from the edge of beam to the centroid of the FRP

a = the depth of the stress block

M_{Total} = the total moment

M = the moment from Fisher (2009)

P_{Load} = the load the specimen should carry

Table 2.3-Specimen data

	Specimen 1	Specimen 3	Specimen 5
f_{FRP}	105 ksi	105 ksi	105 ksi
w_{FRP}	2 in.	2 in.	2 in.
t_{FRP}	0.02 in.	0.02 in.	0.02 in.
F_{FRP}	8.4 kips	8.4 kips	8.4 kips
d	7.063 in.	7 in.	7 in.
a	4 in.	4 in.	4 in.
M_{FRP}	42.6 kip-in.	42 kip-in.	42 kip-in.
M	160.5 kip-in.	194.37 kip-in.	226.07 kip-in.
M_{Total}	203.1 kip-in.	236.37 kip-in.	268.07 kip-in.
P_{Load}	11.3 kips	13.13 kips	14.9 kips

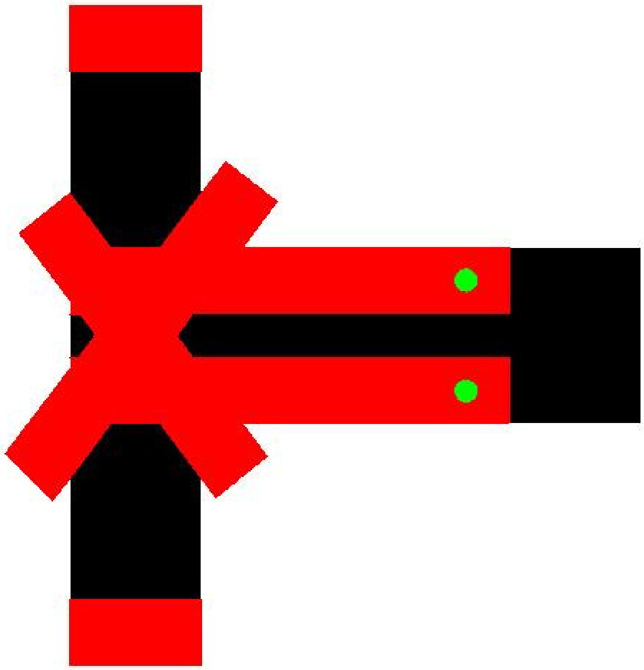

Figure 2.19-FRP layout (Note: Red denotes FRP; green denotes Tapcons® discussed later; black denotes concrete and grout.)

Figure 2.20-FRP cut into pieces in preparation of wrapping

2.3.5 Epoxy

Sikadur® 330 US Epoxy was used to glue the FRP to the surface of the concrete specimens. Sikadur® 330 US Epoxy comes in two parts, Part A and Part B. The epoxy is created once these two parts are combined and mixed (Figure 2.21). The epoxy is applied to the areas of concrete to be wrapped with FRP. Then, the FRP is wrapped and smoothed around the concrete, and another layer of epoxy is applied to the wrap. This causes the wrap to harden as the epoxy dries. This method is called the "Dry Lay-Up Method." Attached in the Appendix is the product data sheet for the epoxy.

Figure 2.21- Sikadur® 330 US Epoxy-Parts A and B before mixing

2.3.6 Protective Equipment

Since Sikadur® 330 US Epoxy is considered a hazardous material, special safety precautions had to be taken to avoid harm. NIOSH respirators, special gloves, goggles, and a plastic suit had to be worn to keep the epoxy from touching skin (Figure 2.22).

Figure 2.22-Protective equipment

2.3.7 Tapcons®

Tapcons® are self-tapping screws. The screws were used to help anchor and hold the FRP in place along the sides of the beams thus avoiding slippage. Holes for the screws were drilled approximately 12 inches from the face of the column and 2 to 3 inches from both edges of the beam. Variations in the locations of the holes occurred due to avoiding longitudinal and traverse steel (Figure 2.23). The retrofitted specimens prior to testing are shown in Figures 2.24 and 2.25.

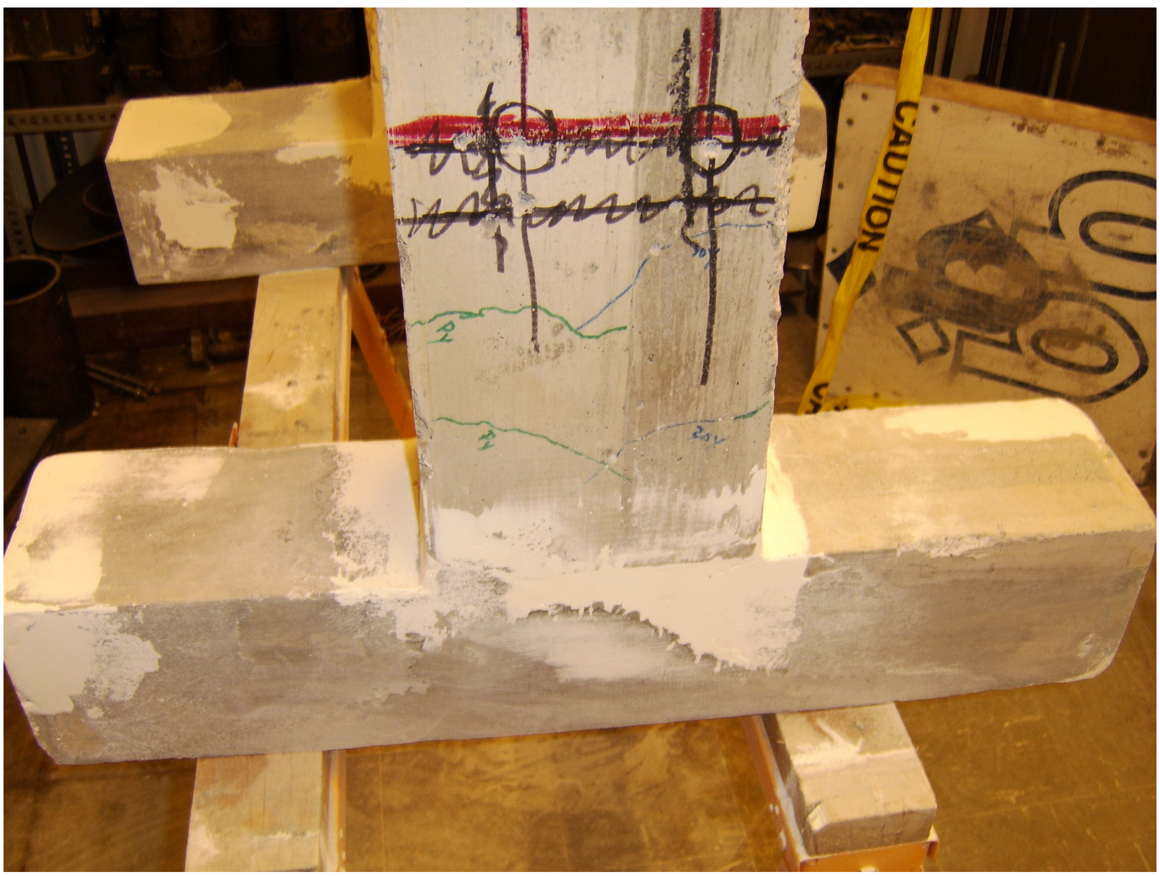

Figure 2.23-Hole locations for two Tapcons® screws

Figure 2.24-Finished retrofitted specimens

Figure 2.25-Second picture of finished retrofitted specimens

CHAPTER 3: TESTING PROCEDURE

3.1 Introduction

In this chapter, the testing apparatus and procedure are described. The specimens were tested following the same procedure established by Fisher (2009). Again, refer to Fisher (2009) for more details regarding the testing apparatus and procedure.

3.2 Testing Apparatus

The testing station used for this research project was located in Hitchcock Hall on the campus of The Ohio State University and is owned by the Civil Engineering Department. Fisher had completed extensive improvements to the testing station to complete his research project.

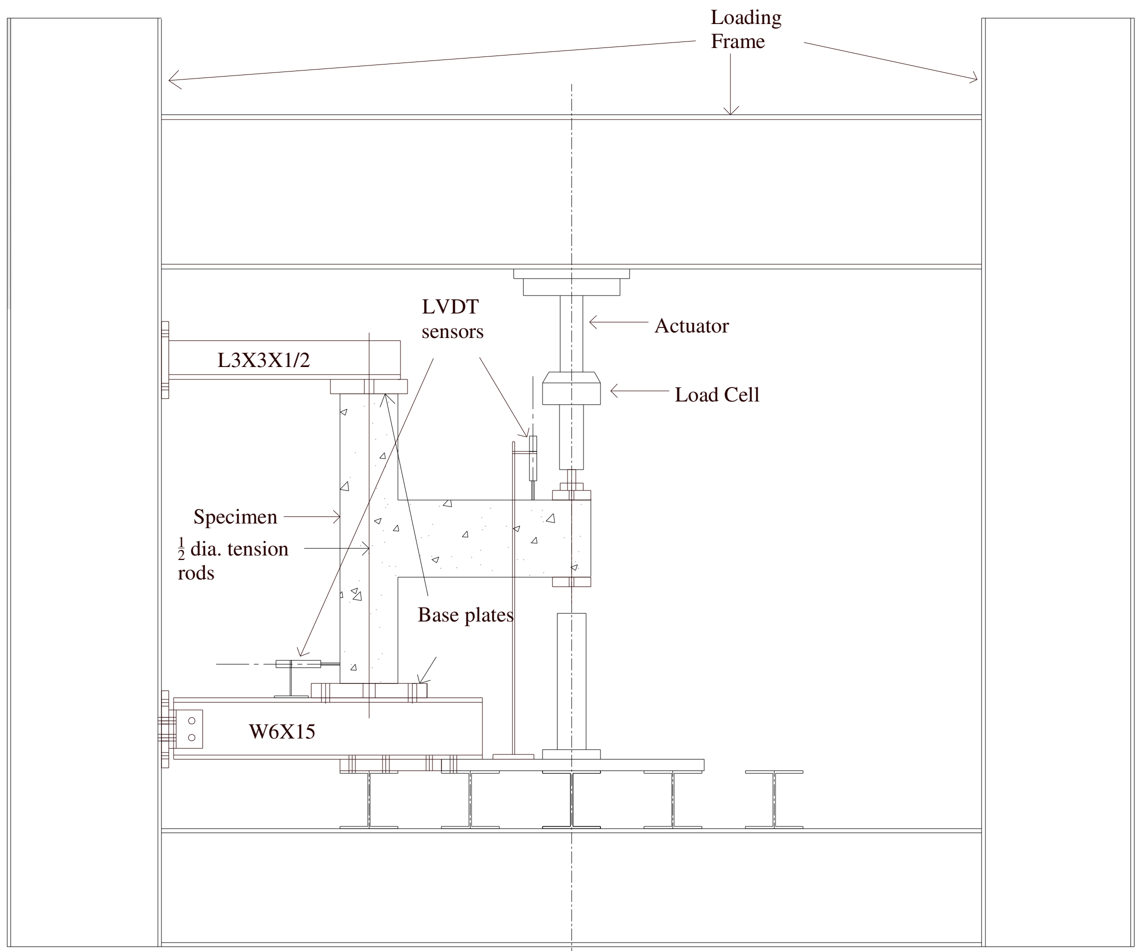

Figure 3.1-Diagram of Testing Station (Fisher 2009)

3.3 Testing Procedure

The three specimens were tested under reverse cyclic loading applied at the tip of the cantilever beam, as specified by Fisher (2009). Reverse cyclic loading involves alternating the specimen between upward and downward deflections. Upward deflections are positive; downward deflections are negative. The load and displacement values specified in Tables 3.1 through 3.3 were calculated by Fisher (2009) and were applied on the original specimens. The same loading scheme is applied on the strengthened specimens in this research. In the tables P_y is the load applied at the tip of the beam corresponding to the first yielding in the longitudinal beam reinforcement. Δ_y is the displacement corresponding to the first longitudinal steel yielding in the beam.

The axial load on the columns was similar to Fisher (2009). Fisher (2009) found the strain in each ½ in. diameter tension rod by using strain gauges. However, strain gauges were not used for this research project, but the axial load of the columns should be similar to Fisher (2009) since his procedure for tightening the nuts on the tension rods was followed.

Table 3.1-Loading schedule for Specimen 1 (Fisher 2009)

Cycle No.	Load Magnitude	Load (kips)
1	P_y /10	0.70
2	P_y /4	1.73
3	P_y /2	3.45
4	3/4 P_y	5.18
5	P_y	6.91
	Displacement Magnitude	Displacement (in.)
6	$2\Delta_y$	0.40
7	$3\Delta_y$	0.60
8	$4\Delta_y$	0.80
9	$6\Delta_y$	1.20
10	$8\Delta_y$	1.60

Table 3.2-Loading schedule for Specimen 3 (Fisher 2009)

Cycle No.	Load Magnitude	Load (kips)
1	P_y /10	1.06
2	P_y /4	2.65
3	P_y /2	5.29
4	3/4 P_y	7.94
5	P_y	10.59
	Displacement Magnitude	Displacement (in.)
6	$2\Delta_y$	1.40
7	$3\Delta_y$	2.10
8	Maximum	2.90

Table 3.3-Loading schedule for Specimen 5 (Fisher 2009)

Cycle No.	Load Magnitude	Load (kips)
1	P_y /10	1.23
2	P_y /4	3.07
3	P_y /2	6.13
4	3/4 P_y	9.2
5	P_y	12.26

CHAPTER 4: TEST RESULTS

4.1 Introduction

In this chapter, the test results are presented, discussed, and compared to Fisher's (2009) results. Pictures are used to describe the damage that occurred to the specimens. Also, load-deflection graphs are included in this chapter to describe the specimens' responses to the loading.

4.2 Specimen 1 Results

Specimen 1 was the strongest specimen tested. All of Fisher's (2009) loading schedule was followed. In addition, a deflection of 3 in. was tested on Specimen 1 because it remained unbroken after Fisher's (2009) loading schedule was completed (Table 3.1).

Specimen 1 eventually failed in flexure. At failure, the longitudinal FRP along the side of the beam tore apart near the joint of the specimen (Figures 4.1 through 4.4). The ends of the column remained in tact. The FRP did a very good job of keeping the column ends from chipping and breaking until the very end of testing. The failure of Specimen 1 can be compared to the original failure after Fisher's testing (Figures 4.5 and 4.6). The recorded load versus displacement relations are shown in Figures 4.7 through 4.17 for each load cycle defined in Table 3.1.

Figure 4.1-Specimen 1 failure

Figure 4.2-Specimen 1 failure

Figure 4.3-Specimen 1 failure

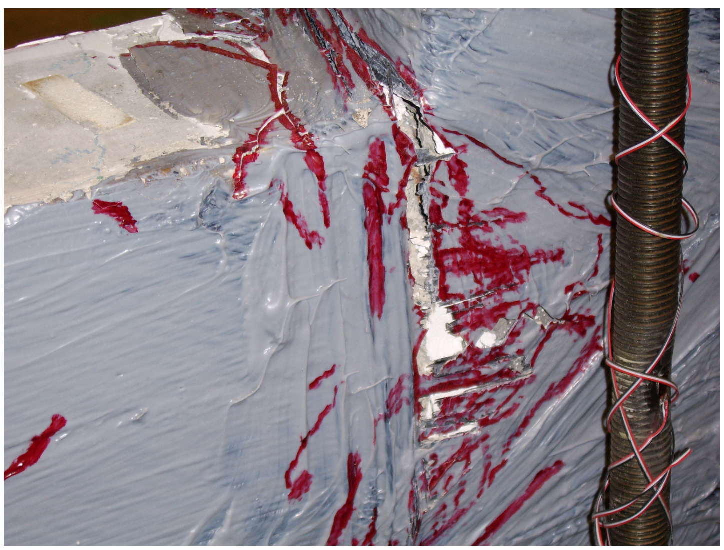

Figure 4.4-Specimen 1 failure

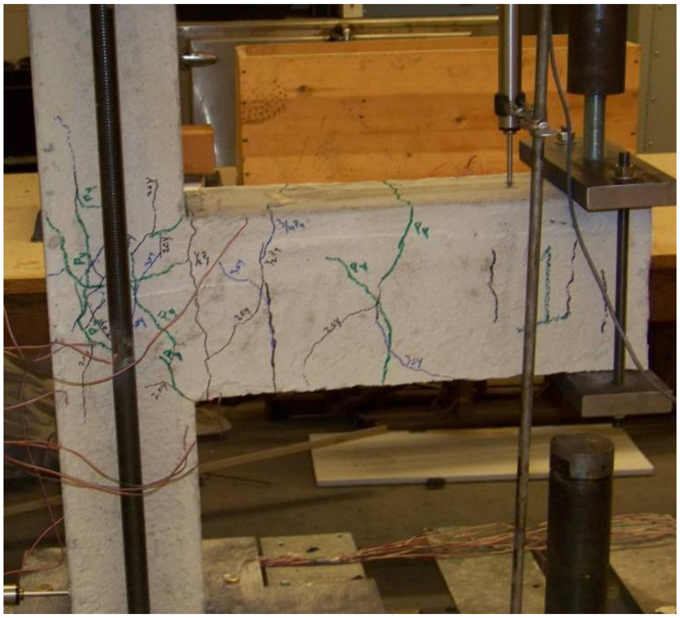

Figure 4.5-Specimen 1 after the $4\Delta_y$ load cycle (Fisher 2009)

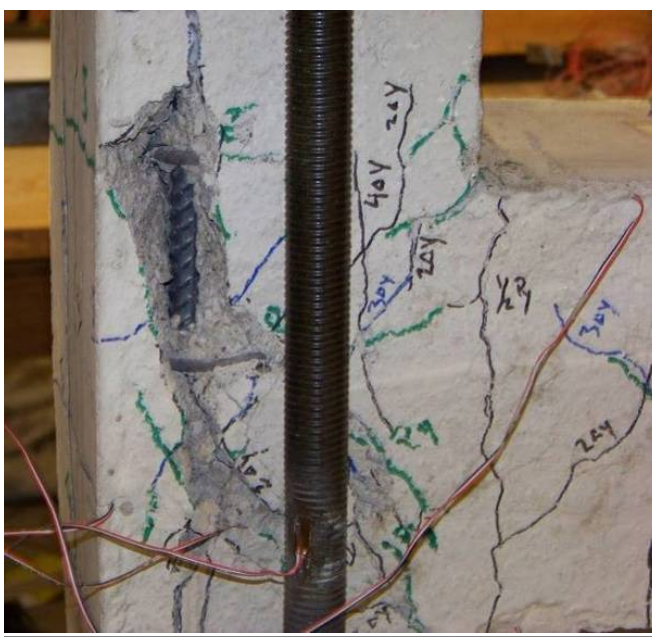

Figure 4.6-Specimen 1 joint damage at failure (Fisher 2009)

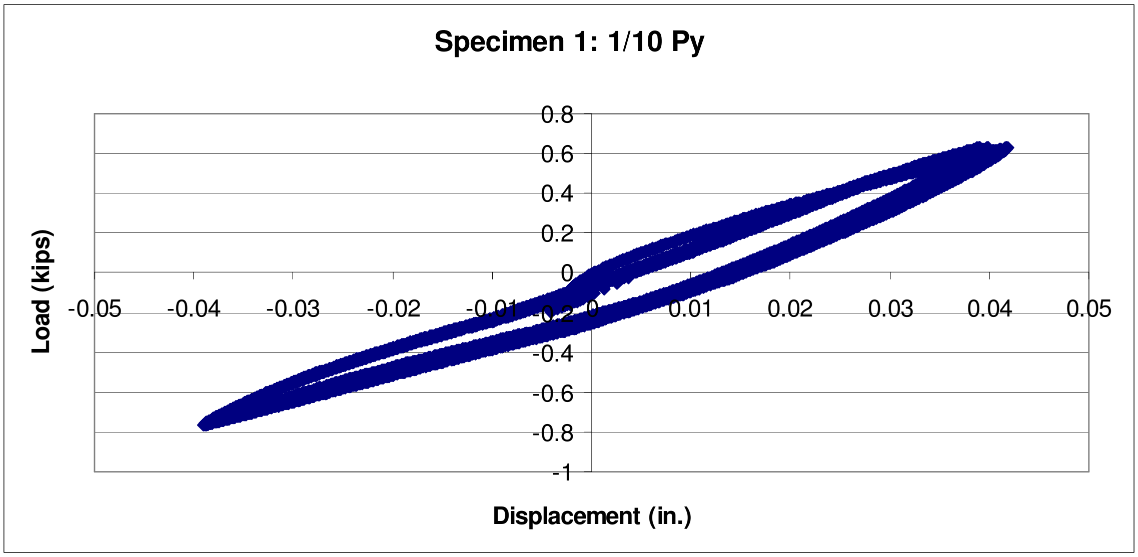

Figure 4.7-Specimen 1 1/10 Py Load-Displacement Graph

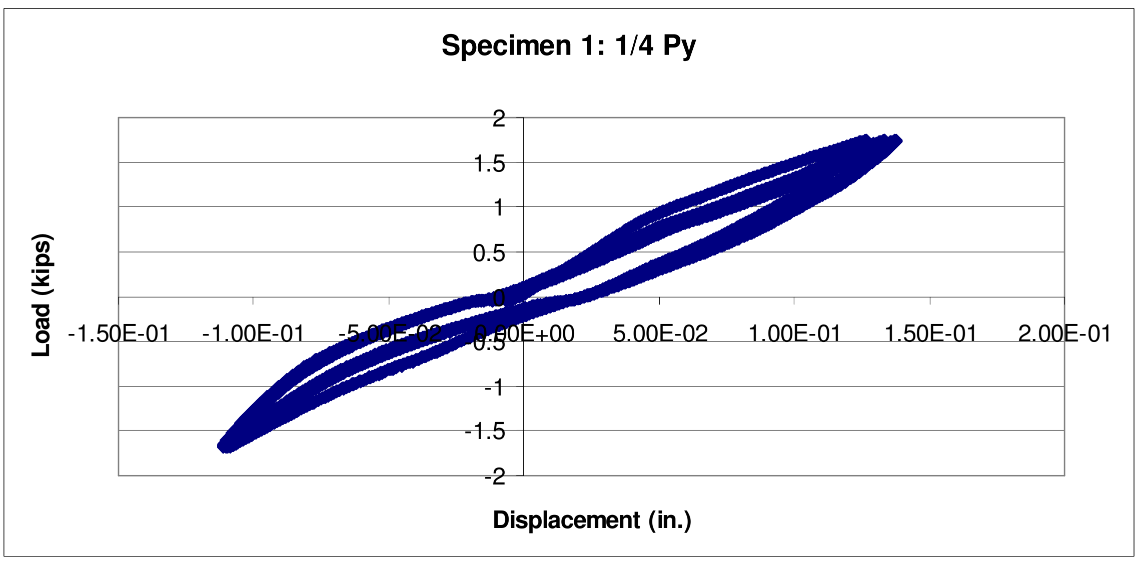

Figure 4.8-Specimen 1 1/4 Py Load-Displacement Graph

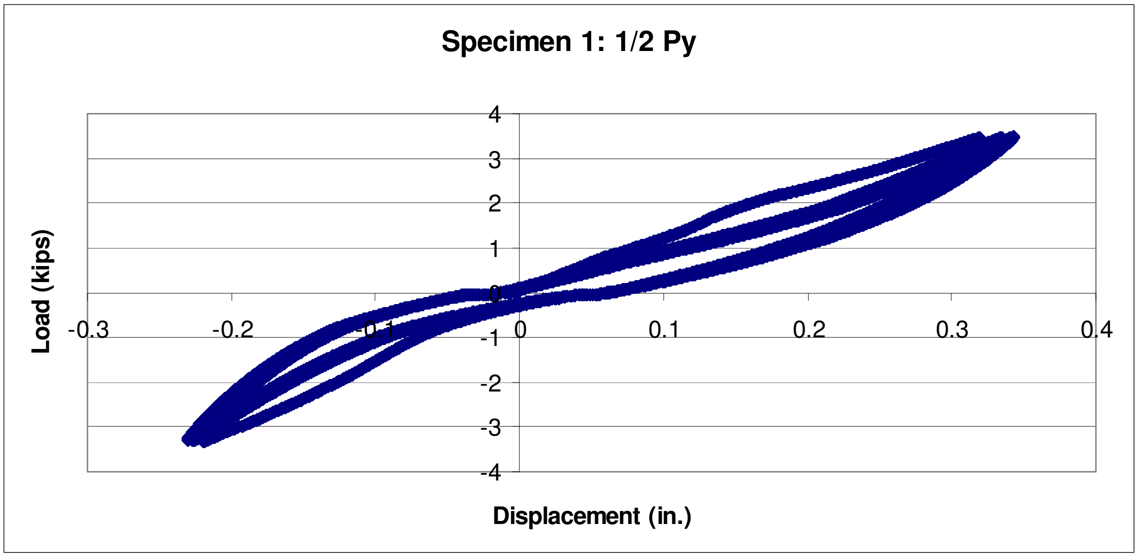

Figure 4.9-Specimen 1 1/2 Py Load-Displacement Graph

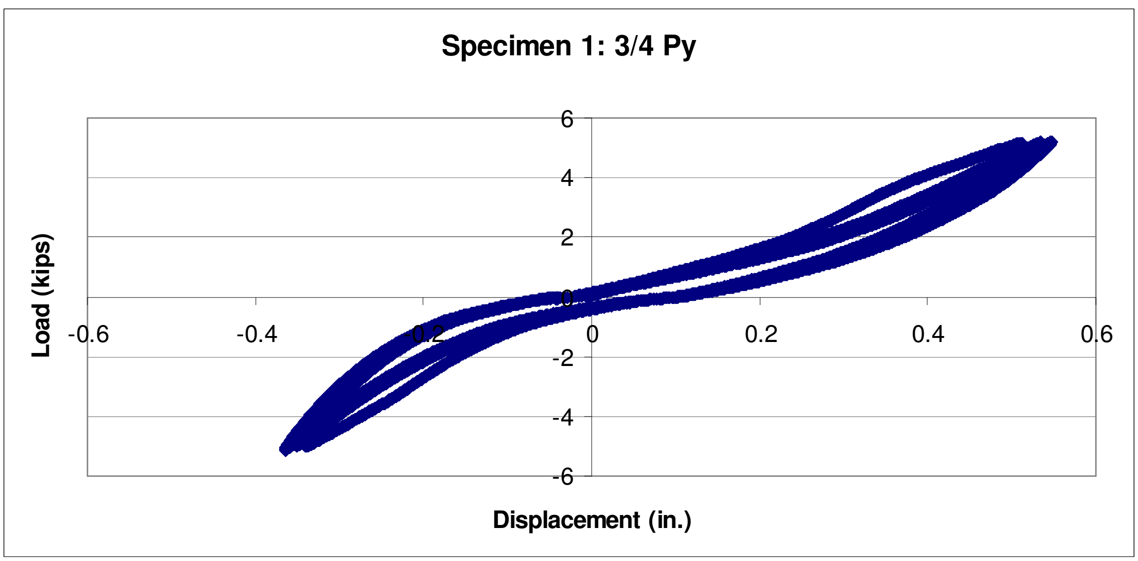

Figure 4.10-Specimen 1 3/4 Py Load-Displacement Graph

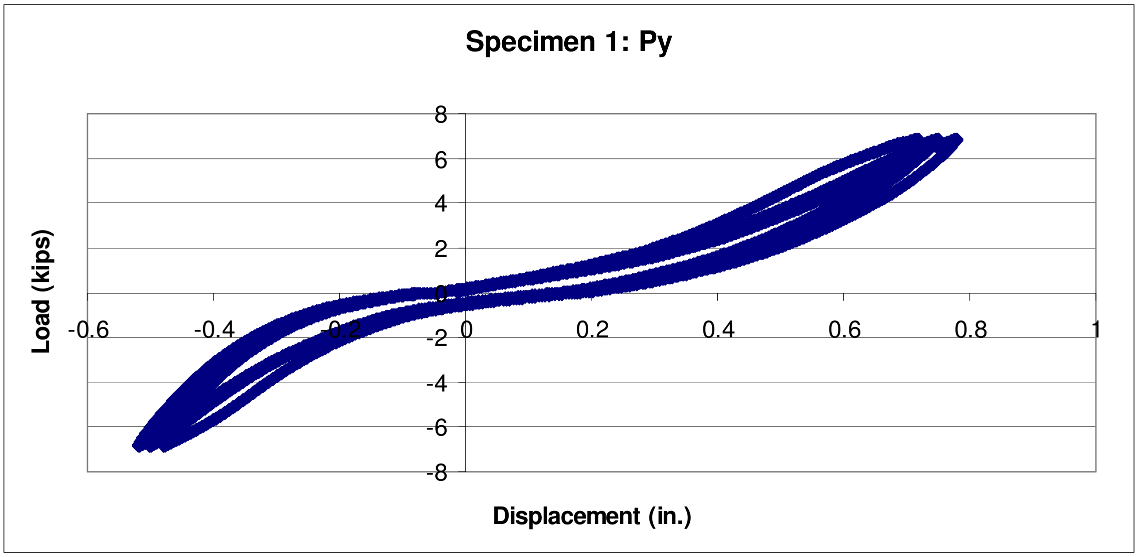

Figure 4.11-Specimen 1 Py Load-Displacement Graph

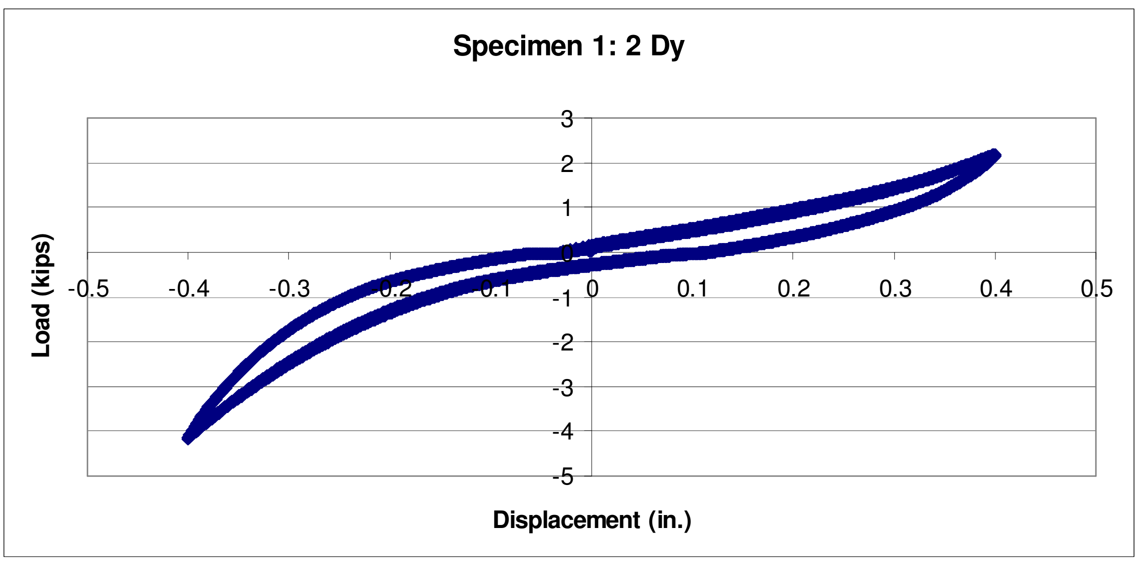

Figure 4.12-Specimen 1 Dy Load-Displacement Graph

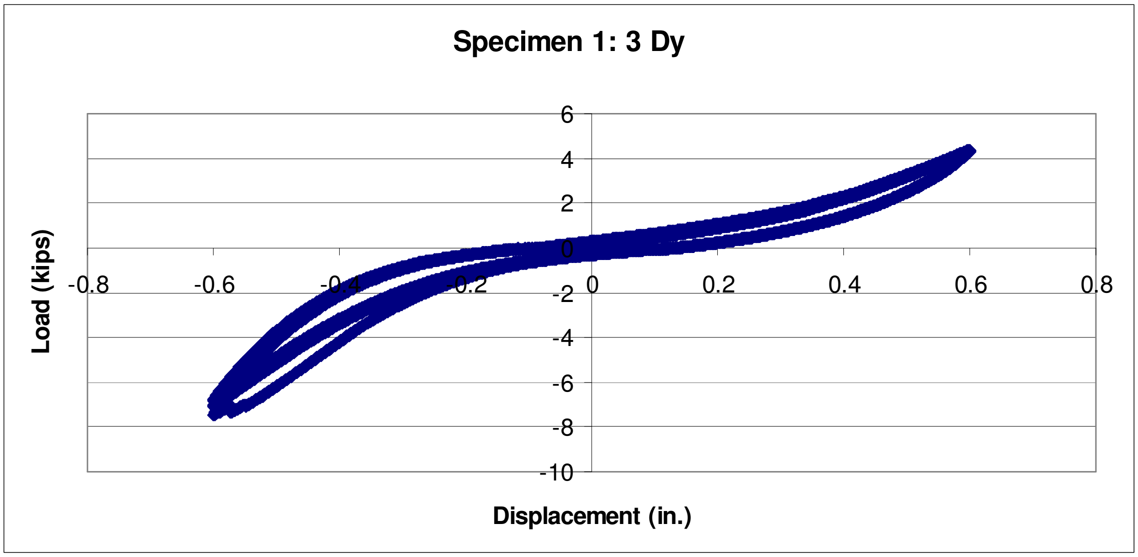

Figure 4.13-Specimen 1 3 Dy Load-Displacement Graph

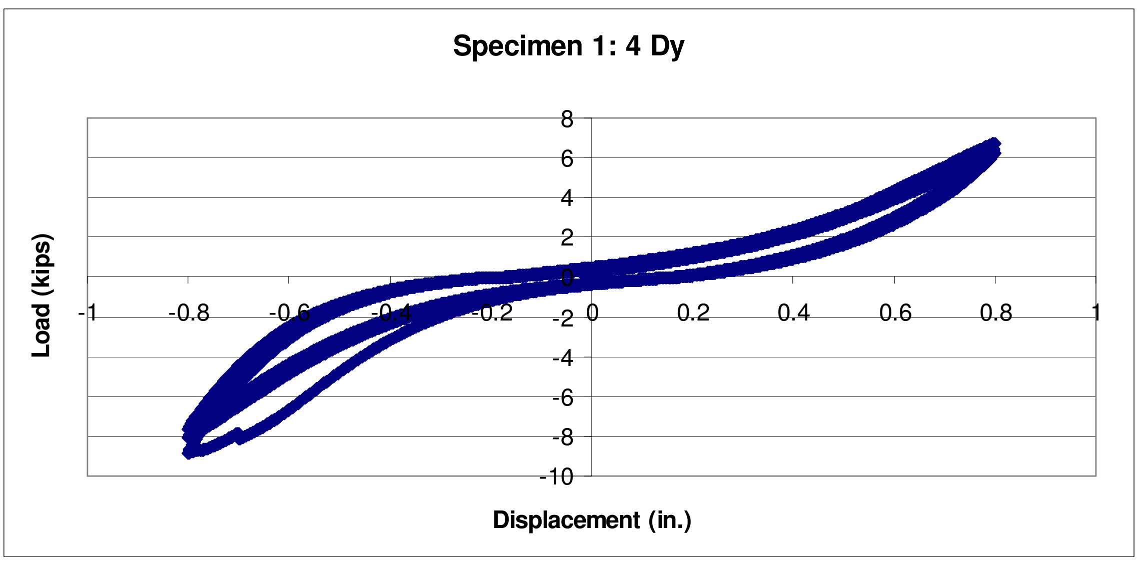

Figure 4.14-Specimen 1 4 Dy Load-Displacement Graph

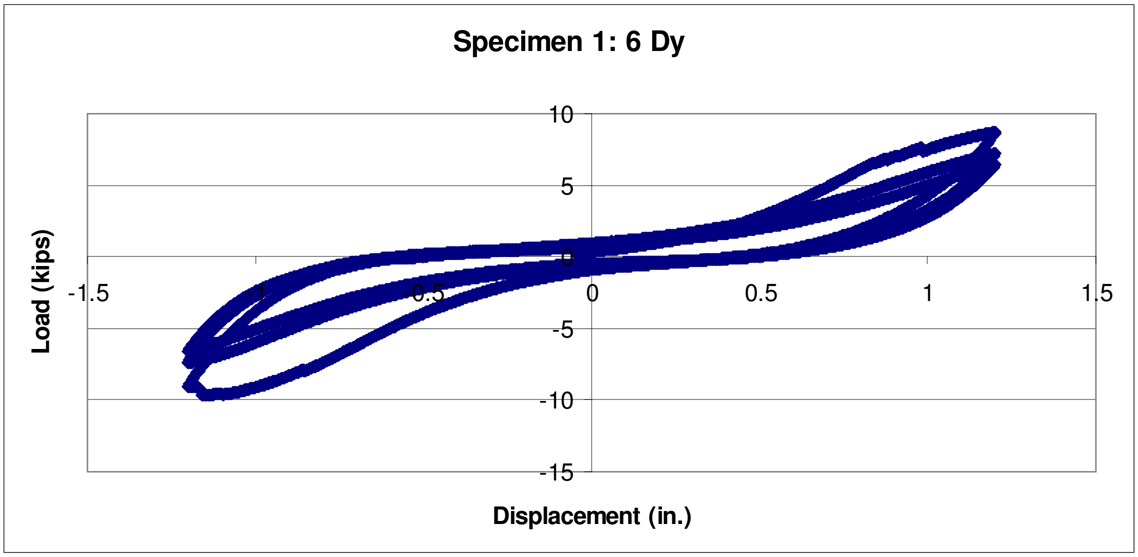

Figure 4.15-Specimen 1 6 Dy Load-Displacement Graph

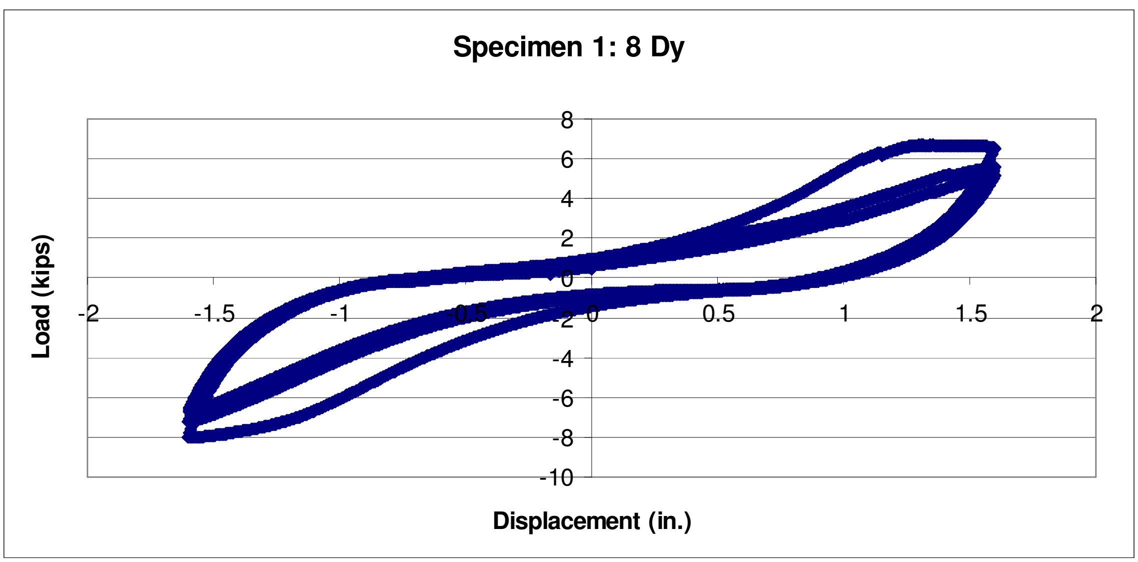

Figure 4.16-Specimen 1 8 Dy Load-Displacement Graph

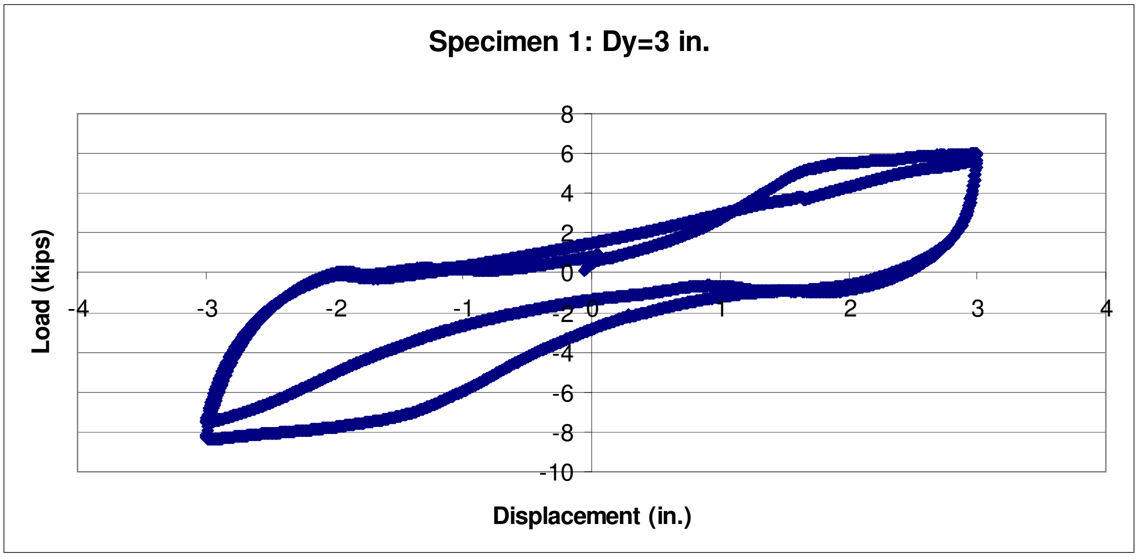

Figure 4.17-Specimen 1 Dy=3 in. Load-Displacement Graph

4.3 Specimen 3 Results

Specimen 3 failed in the middle of the Py cycle of Fisher's (2009) loading schedule and failed in flexure. The longitudinal FRP along the side of the beam tore apart near the joint of the specimen (Figures 4.18-4.21). The ends of the column remained in tact. The FRP did a very good job of keeping the column ends from chipping and breaking. Specimen 3's damage can be compared to the original damage after Fisher's testing (Figures 4.22 and 4.23). Refer to Figures 4.24 through 4.28 for load versus displacement graphs.

Figure 4.18-Specimen 3 failure

Figure 4.19-Specimen 3 failure

Figure 4.20-Specimen 3 failure

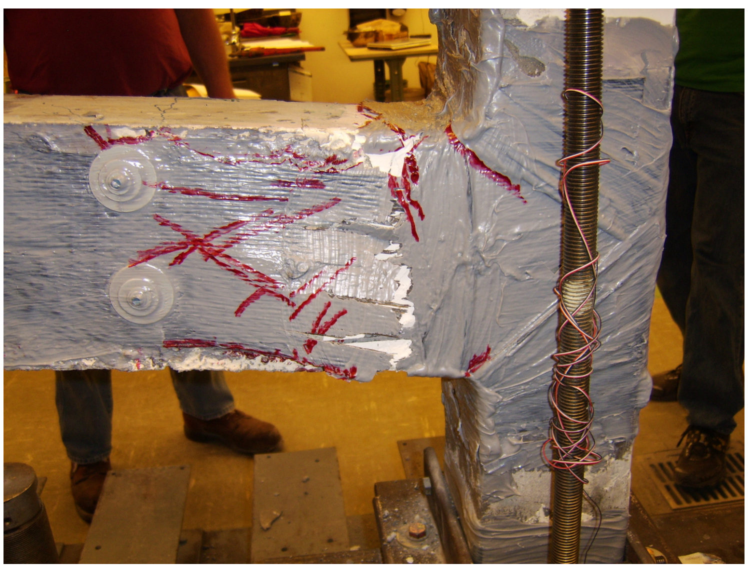

Figure 4.21-Specimen 3 failure

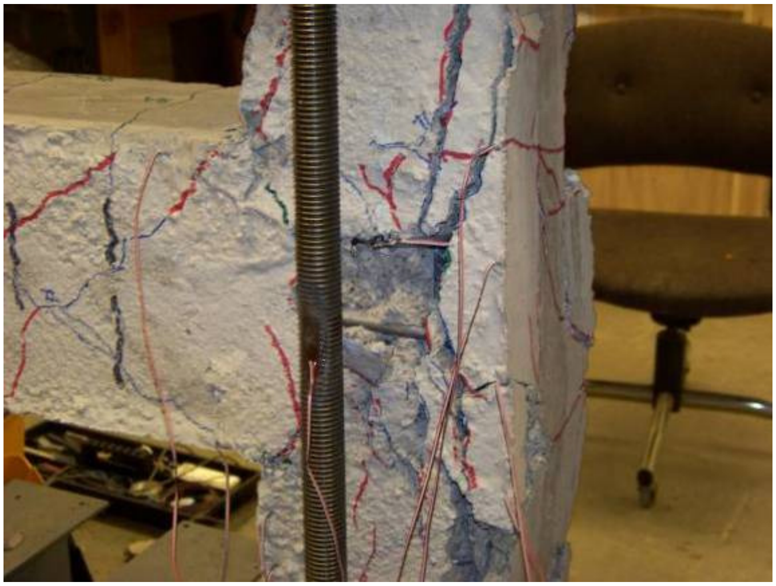

Figure 4.22-Specimen 3 joint region at failure (Fisher 2009)

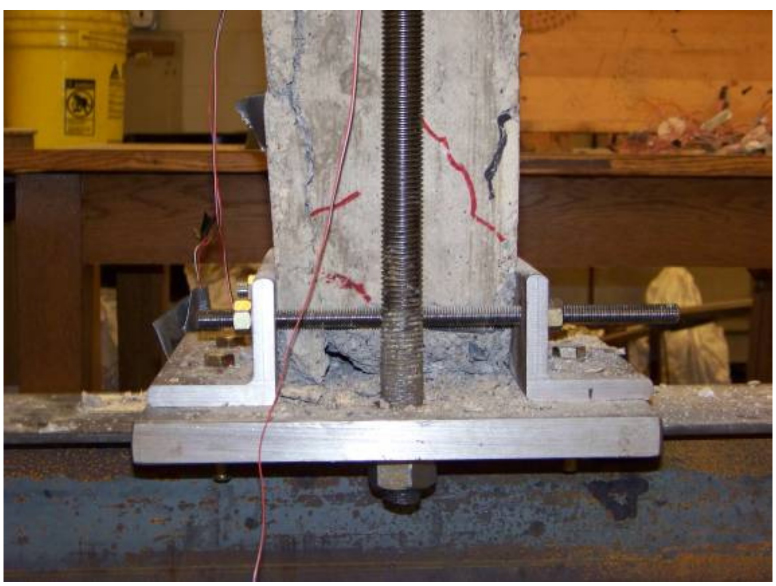

Figure 4.23-Specimen 3 column base at failure (Fisher 2009)

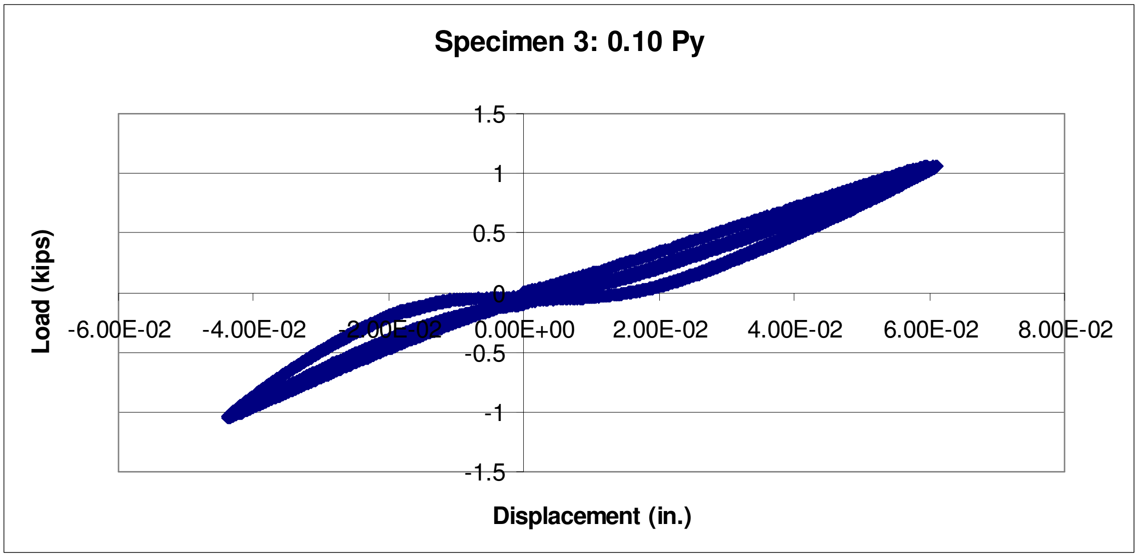

Figure 4.24-Specimen 3 1/10 Py Load-Displacement Graph

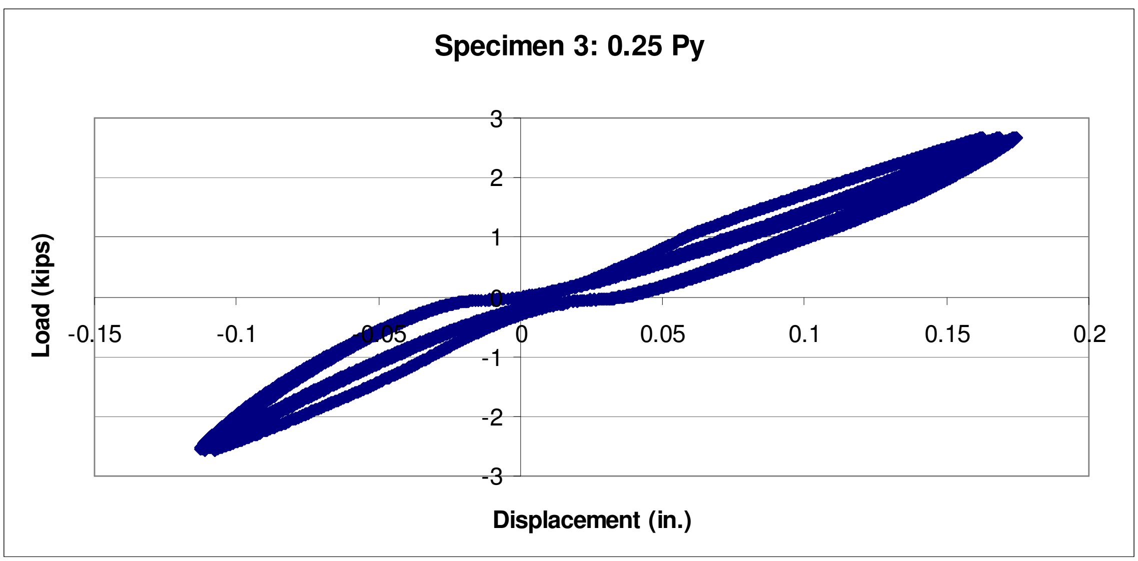

Figure 4.25-Specimen 3 1/4 Py Load-Displacement Graph

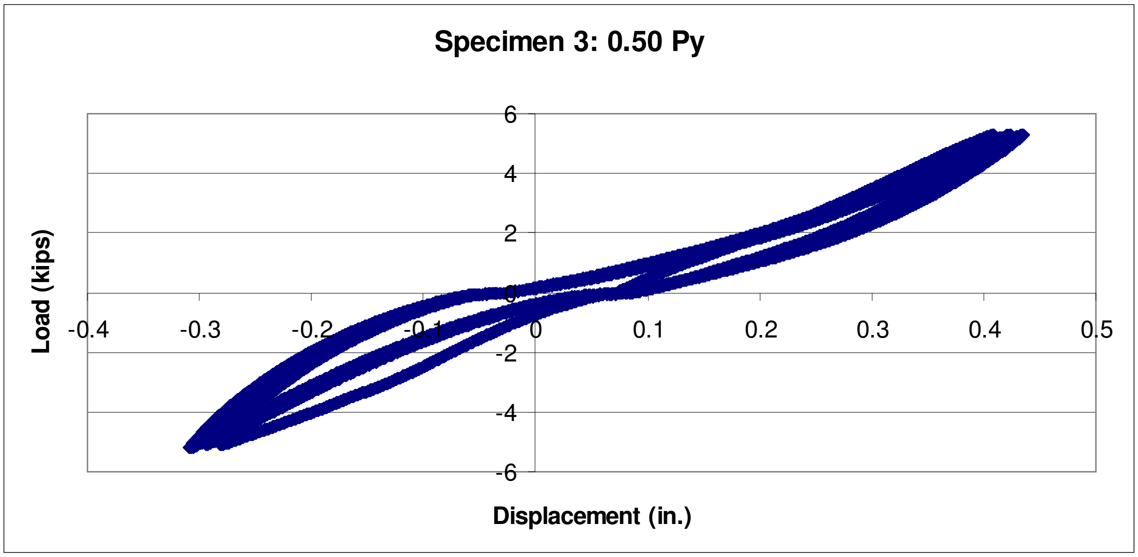

Figure 4.26-Specimen 3 1/2 Py Load-Displacement Graph

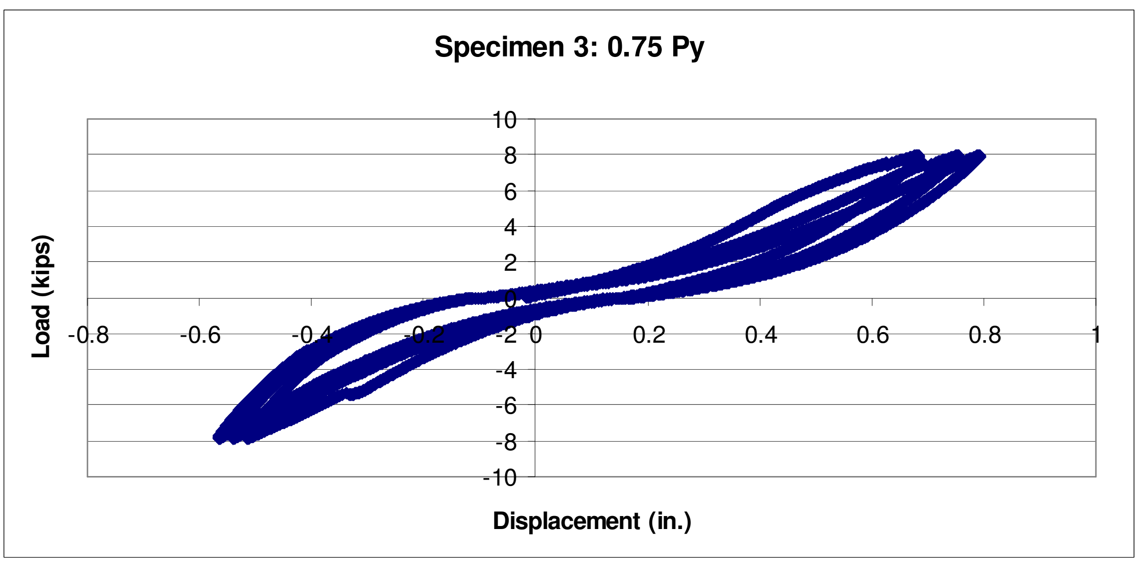

Figure 4.27-Specimen 3 3/4 Py Load-Displacement Graph

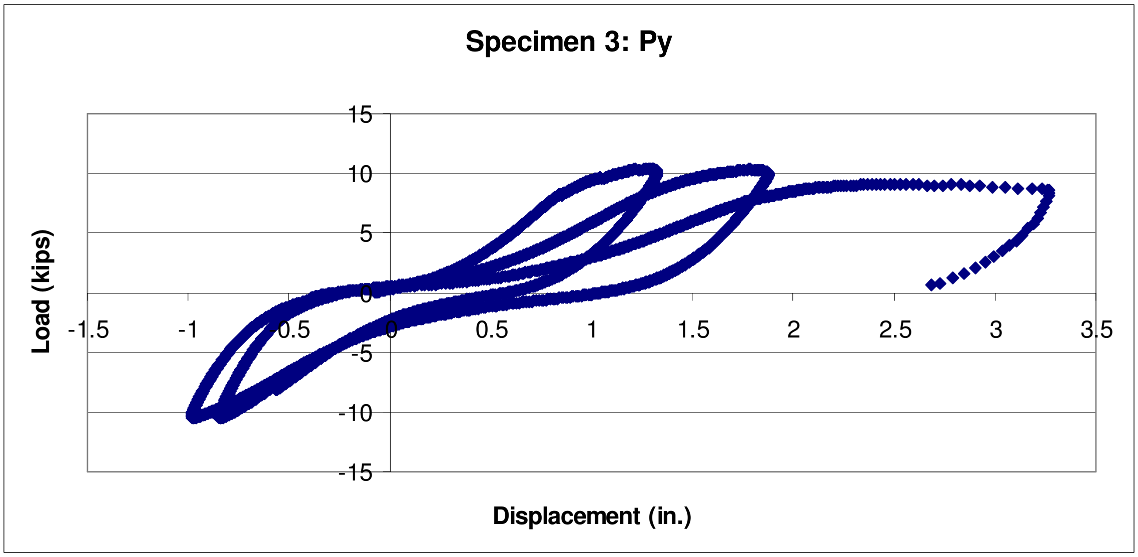

Figure 4.28-Specimen 3 Py Load-Displacement Graph

4.4 Specimen 5 Results

Specimen 5 failed in the middle of the 3/4 Py cycle of Fisher's (2009) loading schedule and failed in flexure. The longitudinal FRP along the side of the beam tore apart near the joint of the specimen (Figures 4.29 through 4.33). The ends of the column remained in tact. The FRP did a very good job of keeping the column ends from chipping and breaking. Specimen 5's damage can be compared to the original damage after Fisher's testing (Figures 4.34 and 4.35). Refer to Figures 4.36 through 4.39 for load versus displacement graphs.

Figure 4.29-Specimen 5 failure

Figure 4.30-Specimen 5 failure

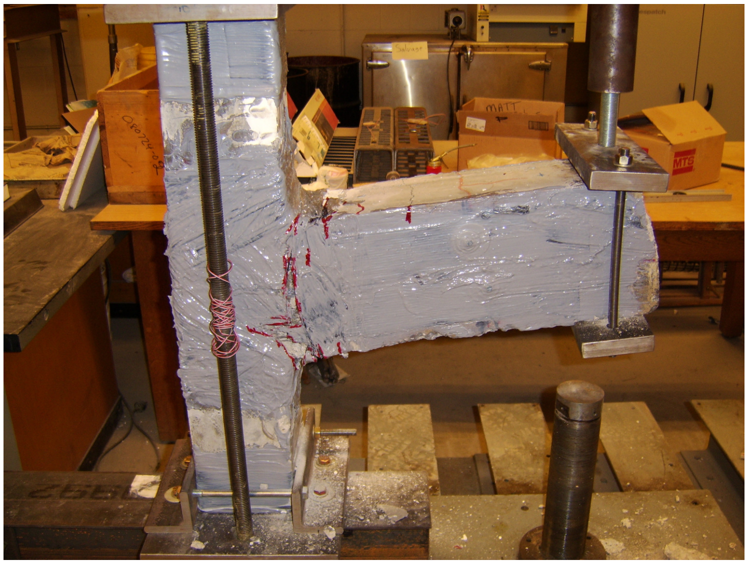

Figure 4.31-Specimen 5 failure

Figure 4.32-Specimen 5 failure

Figure 4.33-Specimen 5 failure

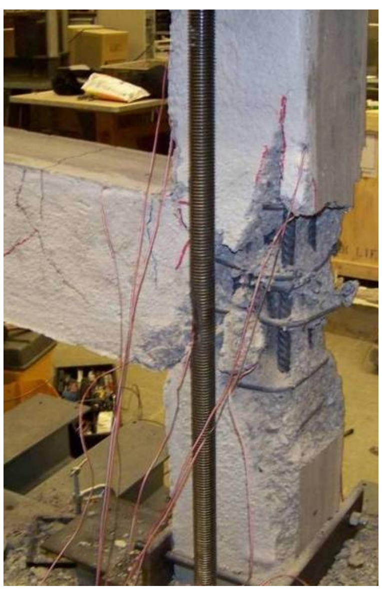

Figure 4.34-Specimen 5 failure after P_y cycle

Figure 4.35-Another picture of Specimen 5 failure after P_y cycle

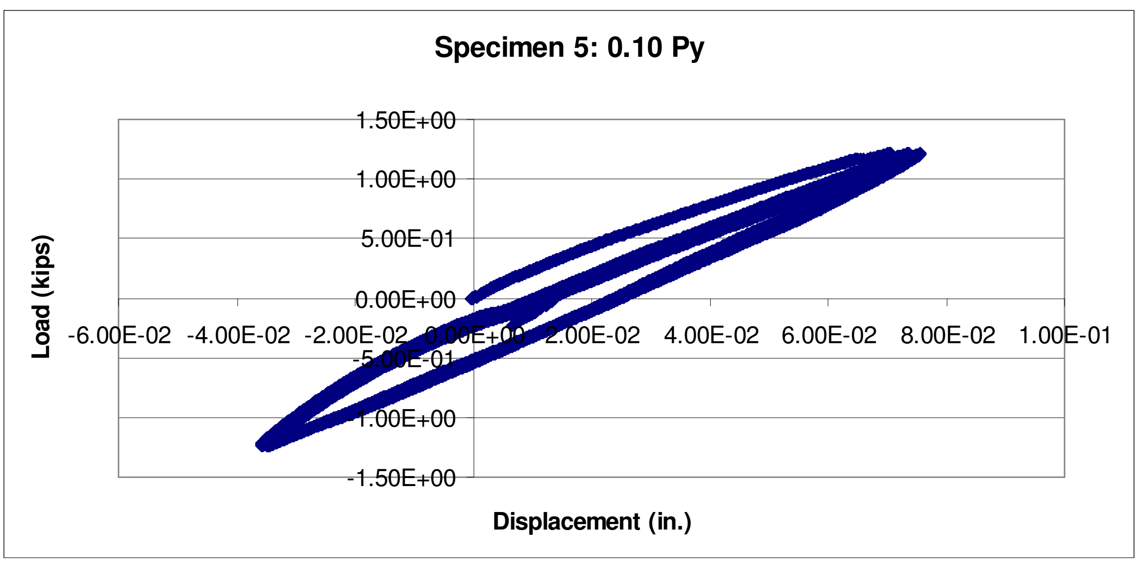

Figure 4.36-Specimen 5 1/10 Py Load-Displacement Graph

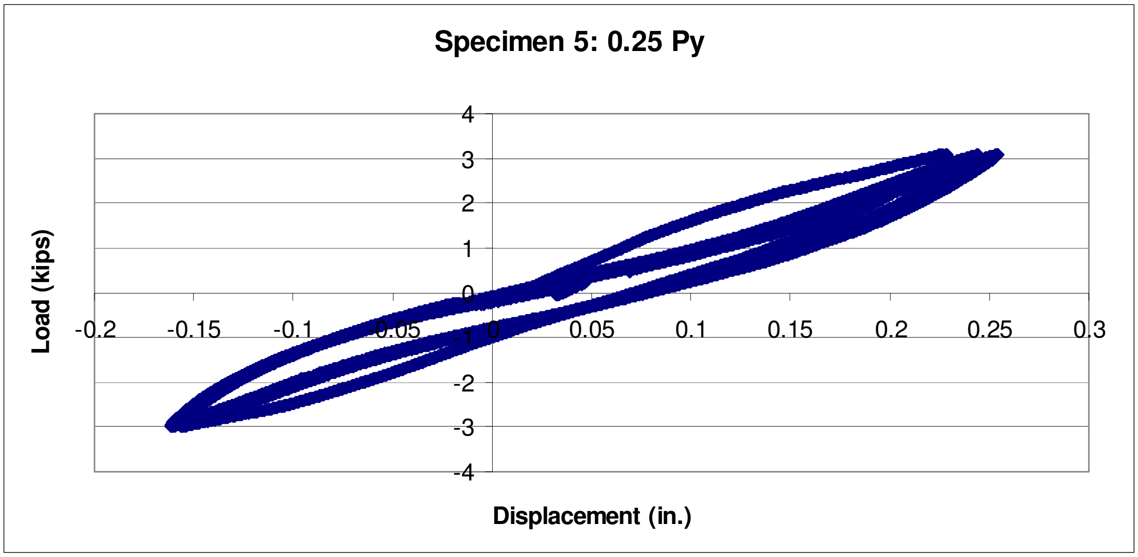

Figure 4.37-Specimen 5 1/4 Py Load-Displacement Graph

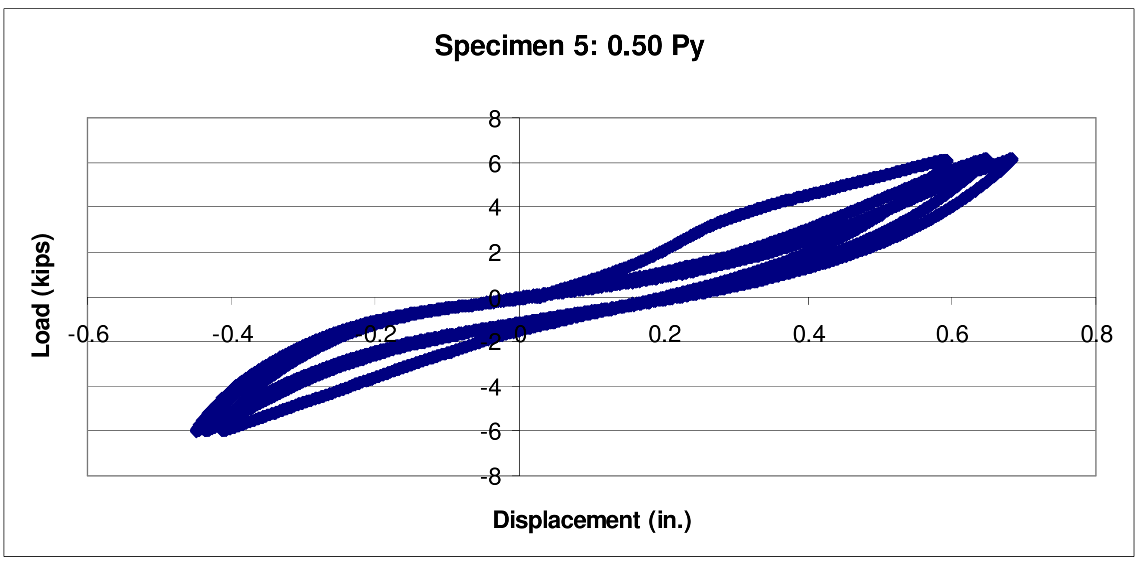

Figure 4.38-Specimen 5 1/2 Py Load-Displacement Graph

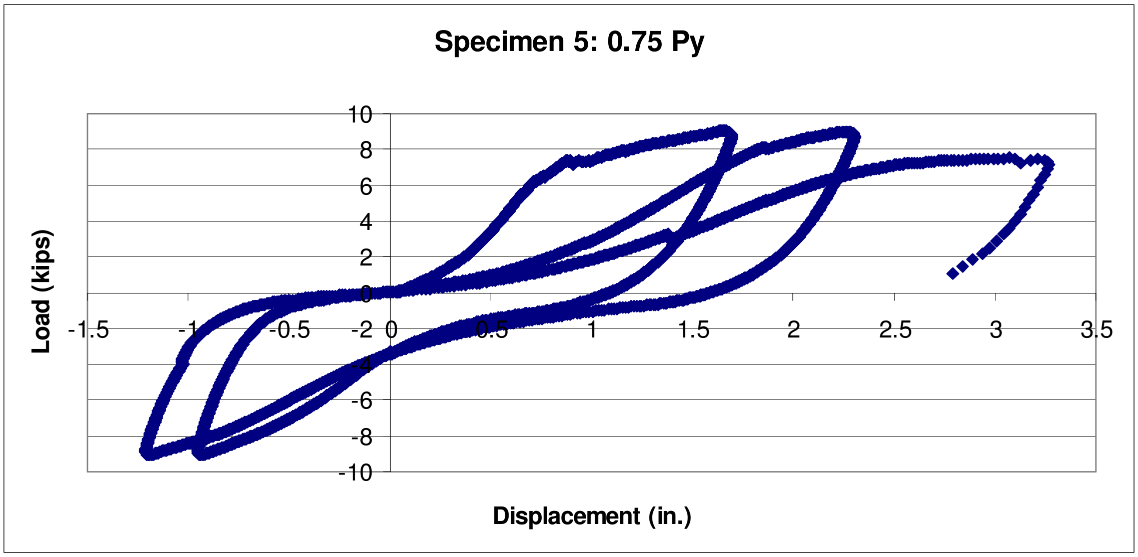

Figure 4.39-Specimen 5 3/4 Py Load-Displacement Graph

CHAPTER 5: CONCLUSIONS

5.1 Summary

This research project involved retrofitting three damaged beam-column joint specimens with non-shrink grout and FRP. Then, these specimens were loaded under cyclic loading similar to the loading they originally experienced during the tests conducted by Fisher (2009). All specimens failed similarly with the longitudinal FRP tearing at the joint. This was due to assuming that the concrete had full-strength when probably it did not. Therefore, more strips of FRP could have been wrapped along the sides of the beam. The reason for assuming the original concrete had full-strength was to ensure that the specimens would fail to see the failure mode. The load actuator could only provide a maximum load of 20 kips. Thus, too many layers of FRP would create more strength than could be exceeded by the load actuator causing the specimens not to fail. In addition, the FRP did a very good job of confining and keeping the ends of the columns in tact. Refer to Table 5.1 for a comparison of the original and retrofitted specimens' maximum strength and deflection.

Table 5.1-Comparison of original and retrofitted specimens' strength and deflection

Specimen #	Original P_{max} (kips)	Retrofitted P_{max} (kips)	Original Δ_{max} (in.)	Retrofitted Δ_{max} (in.)
1	9.1 -9.5	6.91 -8.34	1.4 -1.4	3.0 -3.0
3	10.5 -10.5	10.41 -10.52	1.8 -2.0	3.27 -0.98
5	12.1 -12.1	9.03 -9.08	1.7 -1.1	3.27 -1.21

Note: Positive numbers indicate values when the load actuator was pulling up on the beam. Negative numbers indicate values when the load actuator was pushing down on the beam.

5.2 Future Recommendations

Although this project provides useful knowledge with FRP, more than likely the specimens that Fisher (2009) tested would not be retrofitted in a practical setting. For example, if the specimens were part of an actual building, the building would most likely be torn down and rebuilt and not retrofitted. Beam-column joints that are partially cracked would be better test specimens for retrofitting. For example, beam-column joints that are loaded until hairline cracks form would be more practical for retrofitting.

In addition, another area of future research could be a way to determine how much strength the semi-cracked concrete has before retrofitting. This would provide a more accurate way of determining how much FRP would be needed to re-strengthen the beam-column.

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APPENDIX

A. DATA SHEETS

B. FRP CALCULATIONS